Chapter 5

GRANULAR PILES

5.1 GENERAL

The Central Plain of Thailand is situated on a flat deltaic-marine deposit with a north-south dimension of about 300 km and an east-west width of about 200 km. The presence of thick deposits of soft Bangkok clay and the effects of ground subsidence due to excessive pumping of groundwater create foundation problems to earth structures and infrastructures, especially at approach embankments (transition units) to bridges and viaducts. Basically, the problem lies in the occurrence of differential movements between the pile-supported bridges and viaducts and the ground-supported approach embankments. To mitigate such a natural hazard, granular piles are proposed as the most appropriate and viable foundation treatment.

Granular piles are composed of compacted sand or gravel inserted into the soft clay foundation by displacement method. The term "granular piles" used in this paper refers to the component of compacted gravel and/or sand piles. It also refers to those known as stone columns. The ground improved by compacted granular piles is termed as composite ground. When loaded, the pile deforms by bulging into the subsoil strata and distributes the stresses at the upper portion of the soil profile rather than transferring the stresses into the deeper layers, thus causing the soil to support it. As a result, the strength and bearing capacity of the composite ground can be increased and the compressibility reduced. In addition, lesser stress concentration is developed on the granular piles. Since the component materials are granular and have higher permeability, granular piles could also accelerate the consolidation settlements and minimize the post construction settlements.

5.2 METHODS OF GRANULAR PILE CONSTRUCTION

Various methods for installation of granular piles have been used all over the world depending on their proven applicability and availability of equipment in the locality. The following common methods will be briefly described with their corresponding references.

5.2.1 Vibro-Compaction Method

The vibro-compaction method is used to improve the density of cohesionless, granular soils using a vibroflot which sinks in the ground under its own weight and with the assistance of water and vibration (Baumann and Bauer, 1974; Engelhardt and Kirsch, 1977). After reaching the predetermined depth, the vibroflot is then withdrawn gradually from the ground with subsequent addition of granular backfill thereby causing compaction. The process is repeated in stages forming a compacted column of granular piles. Figure 5.1 illustrates the steps in vibro-compaction process. The range of grain size distribution of soils suitable for this method is shown in Fig. 5.2. The fines content is limited to not more than 18% passing No. 200 U.S. standard sieves.

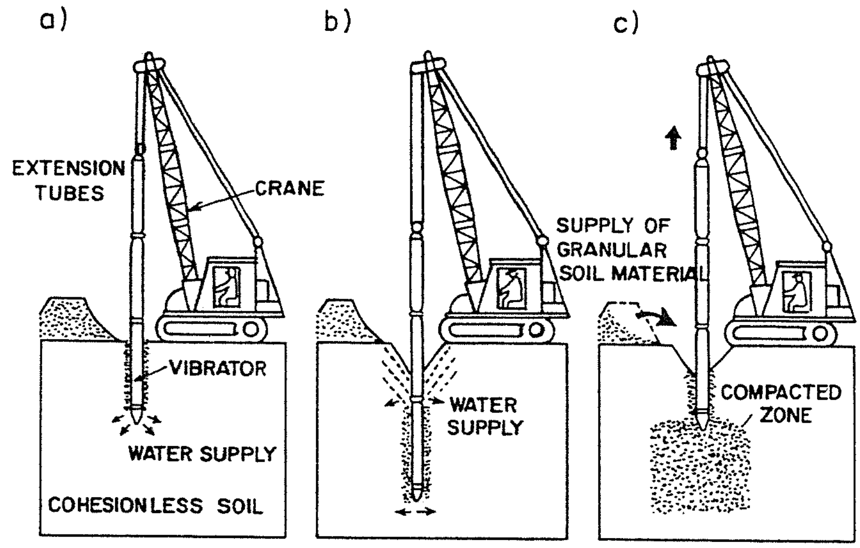

Fig. 5.1 The Vibro-Compaction Process (Baumann and Bauer, 1974)

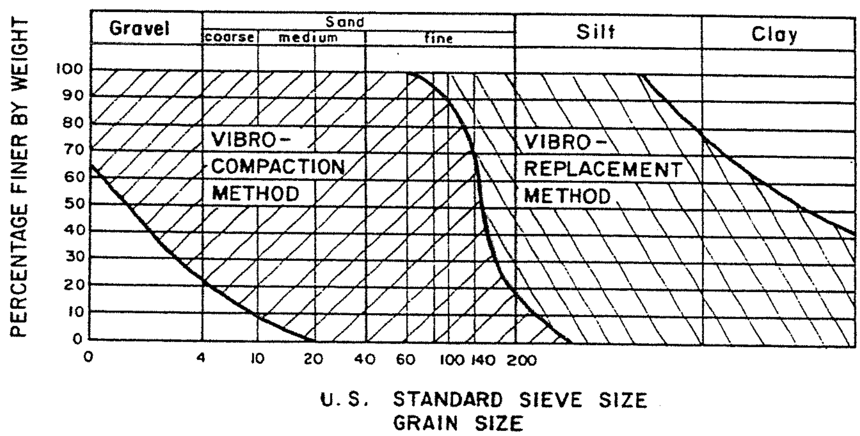

Fig. 5.2 Range of Soils Suitable for Vibro-Compaction Methods (Baumann and Bauer, 1974)

5.2.2 Vibro-Compozer Method

This method is popularized in Japan and is used for stabilizing soft clays in the presence of high groundwater level (Aboshi et al. 1979; Aboshi and Suematsu, 1985; Barksdale, 1981). The installation procedures are illustrated in Fig. 5.3. The resulting pile is usually termed as the sand compaction pile. The sand compaction piles are constructed by driving the casing pipe to the desired depth using a heavy and vertical vibratory hammer located at the top of the pipe. The casing is filled with a specified volume of sand and the casing is then repeatedly extracted and partially redriven using the vibratory hammer starting from the bottom. The process is repeated until a fully penetrating compacted granular pile is constructed.

5.2.3 Cased-Borehole Method

In this method, the piles are constructed by ramming granular materials in the prebored holes in stages using a heavy falling weight (usually of 15 to 20 kN) from a height of 1.0 to 1.5 m (Datye and Nagaraju, 1975; Datye, 1978; Datye and Nagaraju, 1981; Bergado et al. 1984; Ranjan, 1989). The method is a good substitute for vibrator compaction considering its low cost. However, disturbance and subsequent remolding by the ramming operation may limit its applicability to sensitive soils. The method is useful in developing countries utilizing only an indigenous equipment in contrast to the methods described above which require special equipments and trained personnel (Rao, 1982; Ranjan and Rao, 1983). The installation process is illustrated in Fig. 5.4.

Another method of granular pile construction is the vibro-replacement method. This method is discussed in detail in Chapter 3 (Deep Compaction).

5.3 ENGINEERING BEHAVIOR OF COMPOSITE GROUND

The performance of composite ground is best investigated in terms of ultimate bearing capacity, settlement, and general stability. In the following sections, basic relationships of the composite ground as well as failure mechanisms of granular piles on homogeneous soft clay are first described and the ultimate bearing capacity, settlement, and stability of the composite ground based on experimental and analytical studies are then presented.

5.3.1 Basic Relationships

The tributary area of the soil surrounding each granular pile is closely approximated by an equivalent circular area. For an equilateral triangular pattern of granular piles, the equivalent circle has an effective diameter of:

$$D_e = 1.05S \tag{5.1}$$

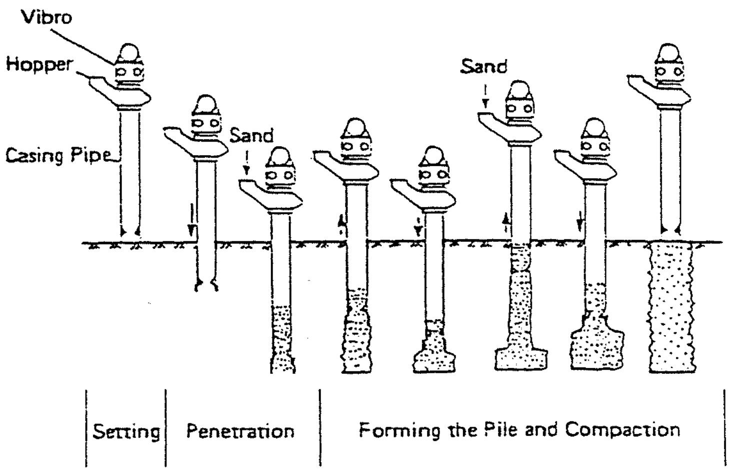

Fig. 5.3 The Vibro-Composer Method (Aboshi and Suematsu, 1985)

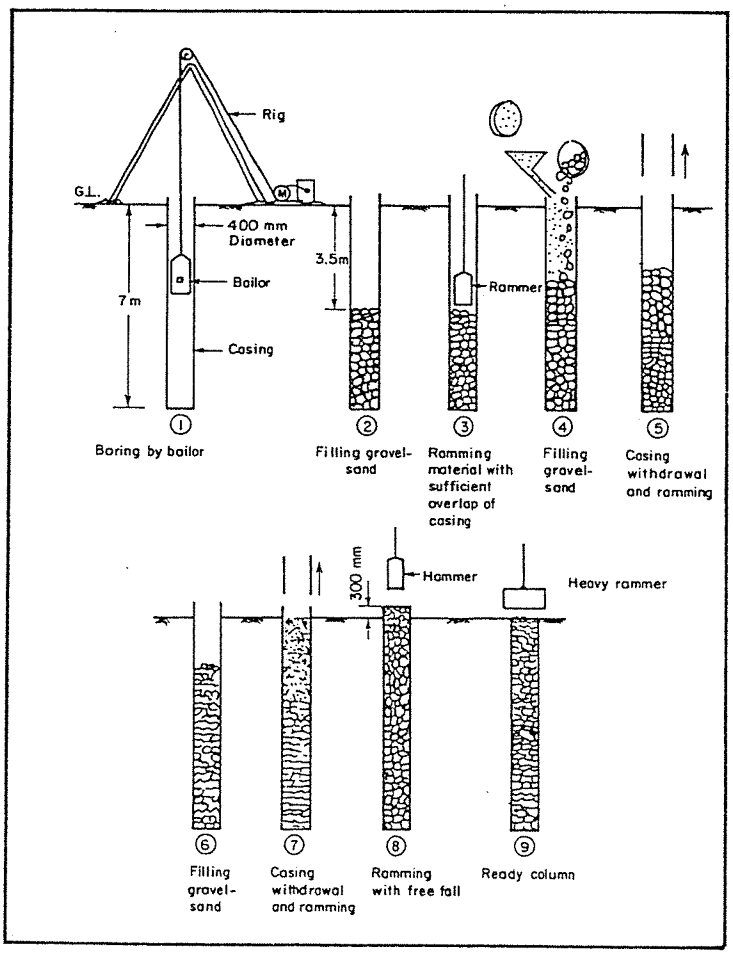

Fig. 5.4 The Cased-Borehole Method (Datye and Nagaraju, 1975)

while for a square pattern,

$$D_e = 1.13S \tag{5.2}$$

where S is the center to center spacing of granular piles. The equilateral triangular pattern gives the most dense packing of granular piles in a given area. The resulting cylinder of composite ground with diameter D_e enclosing the tributary soil and one granular pile is known as the unit cell.

Figure 5.5 illustrates the area replacement factor as well as the stress concentration in the granular pile. The area replacement ratio is defined as the ratio of the granular pile area over the whole area of the equivalent cylindrical unit within the unit cell and expressed as:

$$a_s = \frac{A_s}{A_s + A_c} \tag{5.3}$$

where A_s is the horizontal area of a granular pile, and A_c is the horizontal area of the clayey ground surrounding the pile. The area replacement ratio can also be expressed in terms of the diameter (D) and spacing (S) of the granular pile as follows:

$$a_s = c_1\left(\frac{D}{S}\right)^2 \tag{5.4}$$

where c_1 is a constant depending upon the pattern of granular piles used; for the square pattern $c_1 = \pi/4$ and for the equilateral triangular pattern $c_1 = \pi/(2\sqrt{3})$.

When the composite ground is loaded, studies indicated that concentration of stress occurs in the granular pile accompanied by the reduction in stress which occurs in the surrounding less stiff clayey soil (Fig. 5.5). This can be explained by the fact that, when loaded, the vertical settlement of the granular pile and the surrounding soil is approximately the same, causing the occurrence of stress concentration in the granular pile which is stiffer than the surrounding cohesive or loose cohesionless soil. The distribution of vertical stress within the unit cell can be expressed by a stress concentration factor, n, defined as:

$$n = \frac{\sigma_s}{\sigma_c} \tag{5.5}$$

where σ_s is the stress in the granular pile, and σ_c is the stress in the surrounding cohesive soil. The magnitude of stress concentration also depend on the relative stiffness of the granular pile and the surrounding soil. The variation of stress concentration factor with area replacement ratio compiled by Barksdale and Bachus (1983) ranged from 2 to 5. Meanwhile, Aboshi et al. (1979) and Bergado et al. (1987) obtained a higher value of as much as 9. The higher stress concentration factor obtained by Bergado et al. (1987) was probably due to the high rigidity of the plates used during the load tests. From full scale test embankment observations on soft Bangkok clay at the low area replacement ratio of 0.06, the stress concentration factor of 2 was obtained and was found to decrease to 1.45 with the increasing applied load (Bergado et al. 1988). The average stress, σ, over the unit cell area corresponding to a given area replacement ratio, a_s, is expressed as:

$$\sigma = \sigma_s a_s + \sigma_c(1-a_s) \tag{5.6}$$

The stresses in the pile and the clay using the stress concentration factor are:

$$\sigma_s = \frac{n\sigma}{[1+(n-1)a_s]} = \mu_s\sigma \tag{5.7}$$

$$\sigma_c = \frac{\sigma}{[1+(n-1)a_s]} = \mu_c\sigma \tag{5.8}$$

where μ_s and μ_c are the ratio of stress in the pile and clay, respectively, to the average stress over the unit cell area.

5.3.2 Failure Mechanisms

In practice, granular piles are usually constructed fully penetrating a soft soil layer overlying a firm stratum. It may be constructed also as floating piles with their tips embedded within the soft clay layer. Granular piles may fail individually or as a group. The failure mechanisms for a single pile are illustrated in Figs. 5.6, respectively, indicating the possible failures as: a) bulging, b) general shear, and c) sliding.

5.4 ULTIMATE BEARING CAPACITY OF SINGLE, ISOLATED GRANULAR PILE

For single, isolated granular piles, the most probable failure mechanism is bulging failure. This mechanism develops whether the tip of the pile is floating in the soft soil or fully penetrating and bearing on a firm layer. The lateral confining stress which supports the granular pile is usually taken as the ultimate passive resistance which the surrounding soil can mobilize as the pile bulges outward. Most of the approaches in predicting the ultimate bearing capacity of a single, isolated granular pile has been developed based on the above assumption. Table 5.1 tabulates the different methods to estimate the ultimate bearing capacity corresponding to bulging, general shear, and sliding modes of failure as presented by Aboshi and Suematsu

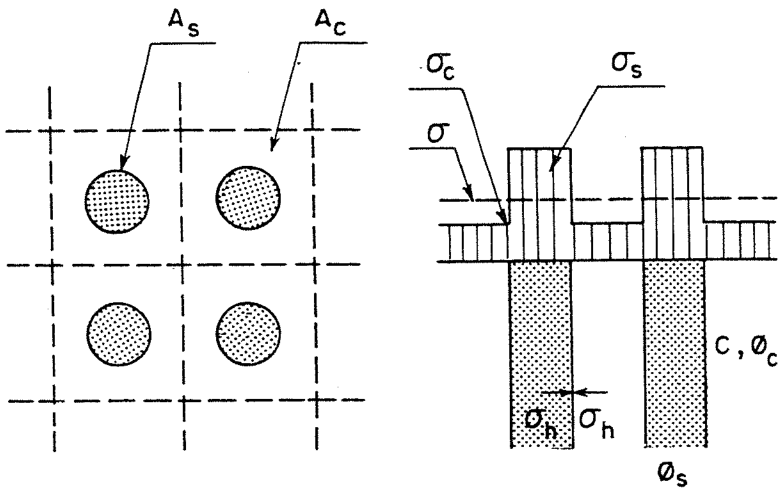

Fig. 5.5 Diagram of Composite Ground

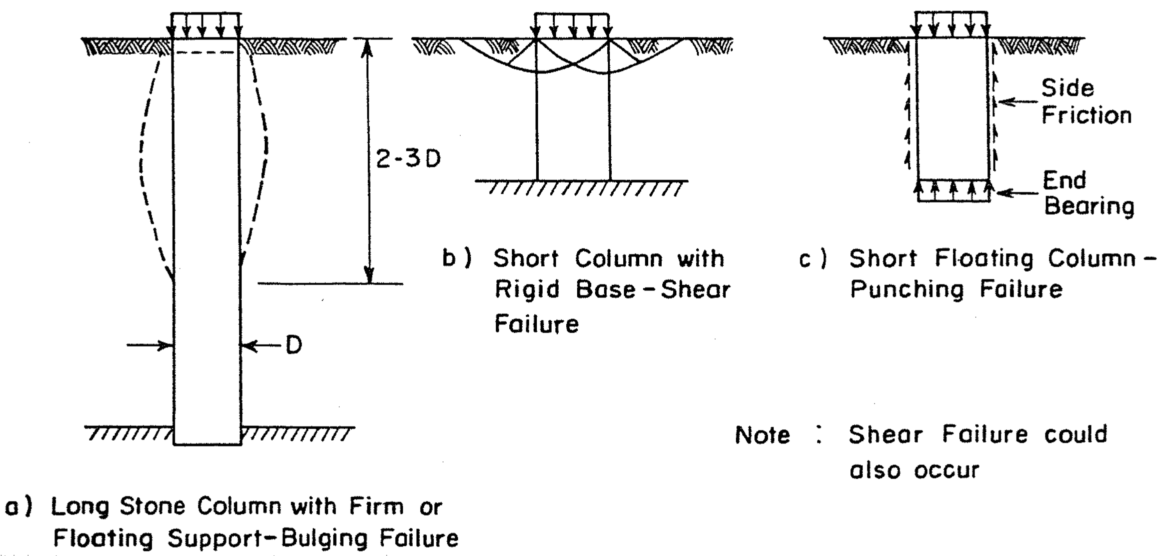

Fig. 5.6 Failure Mechanisms of a Single Granular Pile in a Homogeneous Soft Layer (Barksdale and Bachus, 1983)

Table 5.1 Estimation of Ultimate Bearing Capacity (Aboshi and Suematsu, 1985)

MODE OF FAILURE	DERIVED FORMULA	REFERENCE
Bulging	$q_{ult} = (\gamma_c 2K_{pc} + 2C_o\sqrt{K_{pc}}) \dfrac{1 + \sin\phi_s}{1 - \sin\phi_s}$	Greenwood (1970)
	$q_{ult} = (F'_c C_o + F'_q Q_o) \dfrac{1 + \sin\phi_s}{1 - \sin\phi_s}$	Vesic (1972) Datye and Nagaraju (1975)
	$q_{ult} = (\sigma_{ro} + 4C_o) \dfrac{1 + \sin\phi_s}{1 - \sin\phi_s}$	Hughes and Withers (1974)
	$q_{ult} = \dfrac{1 + \sin\phi_s}{1 - \sin\phi_s} (4C_o + \sigma_{ro} + K_o q_s)(W/B)^2 + [1-(W/B)^2]q_s$	Madhav et al. (1979)
General Shear	$q_{ult} = C_o N_c + (1/2)\gamma_c B N_\gamma + \gamma_c D_f N_q$	Madhav and Vitkar (1978)
	$q_{ult} = 2A_s(K_{pc} q_o + 2C_o\sqrt{K_{pc}}) + (1/K_{as})[3d_s K_{pc}\gamma_c\{1-(3d_s/2L)\}]$	Wong (1975)
	$q_{ult} = (1/2)\gamma_c B\tan^3\psi + 2C_o\tan^2\psi + 2(1-a_s)C_o\tan\psi$ $\psi = 45^o + \dfrac{\tan^{-1}(\mu_s a_s \tan\phi_s)}{2}$	Barksdale and Bachus (1983)
Sliding Surface	$\tau = (1-a_s)C_o + (\gamma_s z + \mu_s\sigma_z)a_s\tan\phi_s\cos^2\theta$ $\mu_s = \dfrac{n}{1 + (n-1)a_s}$	Aboshi et al. (1979)

Note: Refer to APPENDIX for Notations

(1985). A relationship between ultimate bearing capacity and area replacement ratio is shown in Fig. 5.7. The relationship between internal friction angle of granular material, strength of the surrounding clay, and the ultimate bearing capacity of single granular pile is shown in Fig. 5.8.

5.5 ULTIMATE BEARING CAPACITY OF GRANULAR PILE GROUPS

The common method for estimating the ultimate bearing capacity of granular pile groups assumed that the angle of internal friction in the surrounding cohesive soil and the cohesion in the granular pile are negligible. Furthermore, the full strength of both the granular pile and cohesive soil has been mobilized. The pile group is also assumed to be loaded by rigid foundation. The ultimate bearing capacity of granular pile groups as suggested by Barksdale and Bachus (1983) is determined by approximating the failure surface with two straight rupture lines as shown in Fig. 5.9. Assuming the ultimate vertical stress, q_{ult}, and the ultimate lateral stress, σ_3, to be the principal stresses, then the equilibrium of the wedge requires:

$$q_{ult} = \sigma_3 \tan^2\beta + 2c_{avg}\tan\beta \tag{5.9}$$

where:

$$\sigma_3 = \frac{\gamma_c B \tan\beta}{2} + 2c \tag{5.10}$$

$$\beta = 45 + \frac{\phi_{avg}}{2} \tag{5.11}$$

$$\phi_{avg} = \tan^{-1}(\mu_s a_s \tan\phi_s) \tag{5.12}$$

$$c_{avg} = (1 - a_s)\, c \tag{5.13}$$

where γ_c = saturated or wet unit weight of the cohesive soil; B = foundation width; ß = failure surface inclination; c = undrained shear strength within the unreinforced cohesive soil; ϕ_s = angle of internal friction of the granular soil; ϕ_{avg} = composite angle of internal friction; c_{avg} = composite cohesion on the shear surface.

The development of the above approach did not consider the possibility of a local bulging failure of the individual pile. Hence, the approach is only applicable for firm and stronger cohesive soils having an undrained strengths greater than 30-40 kN/m². However, it is useful for approximately determining the relative effects on ultimate bearing capacity design variables such as pile diameter, spacing, gain in shear strength due to consolidation, and angle of internal friction.

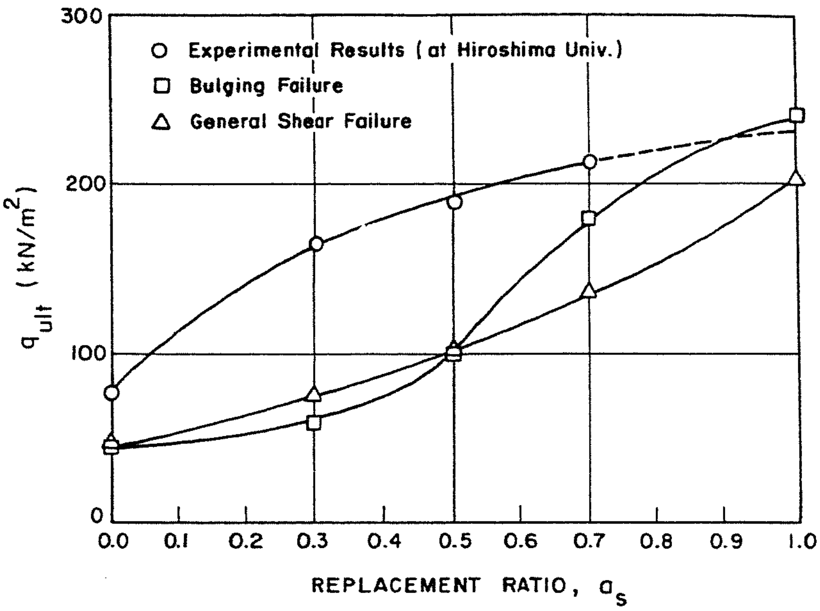

Fig. 5.7 Relationship Between Ultimate Bearing Capacity and Area Replacement Ratio (Aboshi and Suematsu, 1985)

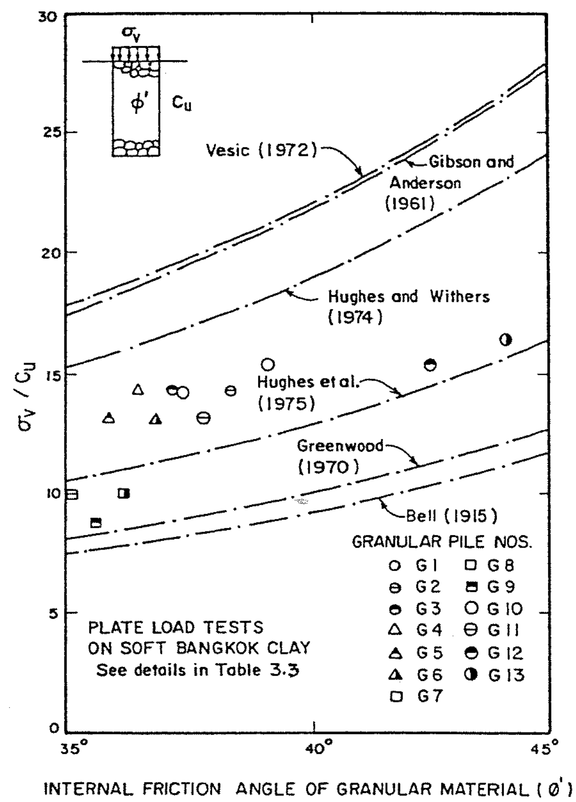

Fig. 5.8 Relationship Between Internal Friction Angle of Granular Material, Strength of Surrounding Clay and Ultimate Bearing Capacity of Single Granular Pile (Bergado and Lam, 1987)

For the case of the soft and very soft cohesive soils, the pile group capacity is predicted using the capacity of a single, isolated pile located within a group and to be multiplied by the number of piles (Barksdale and Bachus, 1983). The ultimate bearing capacity for a single, isolated pile in this case is expressed as:

$$q_{ult} = cN_c^{\prime} \tag{5.14}$$

where N_c = composite bearing capacity factor for the granular pile which ranges from 18 to 22. For the soft Bangkok clay, N_c ranges from 15 to 18 using an initial pile diameter of 25.4 cm with the gravel compacted by a 0.16 ton hammer dropping 0.70 m (Bergado and Lam, 1987).

5.6 SETTLEMENT OF THE COMPOSITE GROUND

Most of the approaches in estimating settlement of the composite ground assumed an infinitely wide, loaded area reinforced with granular piles having a constant diameter and spacing. For this loading condition and geometry, the unit cell idealization is assumed to be valid. The model of a unit cell loaded by a rigid plate is analogous to a one-dimensional consolidation test. Thus, the unit cell is confined by a rigid frictionless wall and the vertical strains at any horizontal level are uniform. Different methods for estimating the settlement of the composite ground are summarized in Table 5.2. The settlement reduction ratio is expressed as:

$$R = \frac{S_t}{S_o} \tag{5.15}$$

where S_t = settlement of the composite ground and S_o = settlement of the unimproved ground. In the case of the equilibrium method, estimation of the settlement of the composite ground is expressed as:

$$S_t = m_v(\mu_c \sigma) \tag{5.16}$$

where m_v = modulus of volume compressibility and H = thickness of layer. The settlement reduction ratio is also expressed as a function of the area replacement ratio (a_s), angle of internal friction of the granular materials (ϕ_s), stress concentration factor, etc. Figure 5.10 shows the relationships between the settlement reduction ratio and the aforementioned parameters based on different methods together with the results from the work of Bergado et al. (1987) on soft Bangkok clay.

5.7 SLOPE STABILITY OF THE COMPOSITE GROUND

Granular piles could be used to increase the stability of slopes and embankments constructed over soft cohesive ground. The method of stability analysis on a composite ground performed exactly in the same manner as for a normal slope stability problem except that stress concentration is considered. When circular rotational failure is expected, the simplified method

Table 5.2 Estimation of Settlement of Composite Ground (Aboshi and Suematsu, 1985)

METHODS	CONTENTS	REFERENCES
Equilibrium Method	$S_t = m_v(\mu_c \sigma)H$ $R = \mu_c = \dfrac{1}{1 + (n-1)a_s}$	Aboshi et al. (1979)
Priebe Method	$\dfrac{1}{R} = 1 + a_s[\dfrac{1/2 + f(\mu,a_s)}{(K_A)_s f(\mu,a_s)} - 1]$ $f(\mu,a_s) = [\dfrac{1-\mu^2}{1-\mu-2\mu^2}][\dfrac{(1-2\mu)(1-a_s)}{1-2\mu+a_s}](K_A)_s = \tan^2(45^o - \dfrac{\phi_s}{2})$	Priebe (1976)
Granular Wall Method	$S_t = RH(1-\mu^2)(1 - \dfrac{\mu^2}{1-\mu^2})\dfrac{\sigma}{E}$ $R = f(a_s,\phi_s,\mu,\sigma/E)$	Van Impe and De Beer (1983)
Incremental Method	$\varepsilon_y = (1-a_s)\dfrac{C_c}{1+e_o}\log_{10}[\dfrac{(P_o)_{vc}+\Delta P}{(P_o)_{vc}}]$ $\Delta P = \dfrac{(\Delta P)^*_{vc}}{1+2K_o}[1 + K + K_o \begin{pmatrix} K \text{ if } K>1 \\ 1 \text{ if } K\leq 1\end{pmatrix}]$ $K = K_o + \dfrac{1}{\varepsilon_v}[\sqrt{\dfrac{1}{1-\varepsilon_v}} - 1]\dfrac{\sqrt{a_s}}{1-\sqrt{a_s}}$ $(\Delta P)^*_{vc} = \dfrac{(\Delta P)^*_v + (P_o)_{vc}a_s - K_o(P_o)_{vc}a_s\tan^2(45+\phi_s/2)}{KFa_s\tan^2(45+\phi_s/2)}$ $R'_p = \dfrac{\varepsilon_v}{\dfrac{C_c}{1+e_o}\log_{10}[\dfrac{(P_o)_{vc}+(\Delta P)^*_v}{(P_o)_{vc}}]}$	Goughnour (1983) Baumann and Bauer (1974) Hughes et al. (1975)
Finite Element Method	$[K_E]\{\Delta\sigma^{(m-1)}\} = \{\{\Delta F_E\} + [K_C^{(m)}]\{\Delta\sigma^{(m)}\} + \{\Delta F_{DN}^{(m)}\}$	alaam and Poulos (1983)

Note: Refer to APPENDIX for Notations

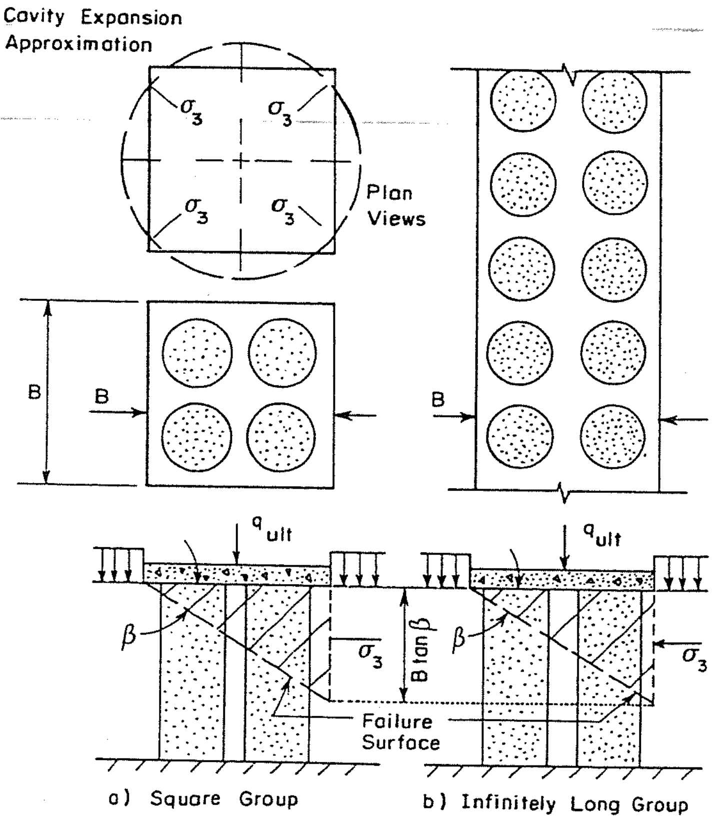

Fig. 5.9 Granular Pile Group Analysis (Barksdale and Bachus, 1983)

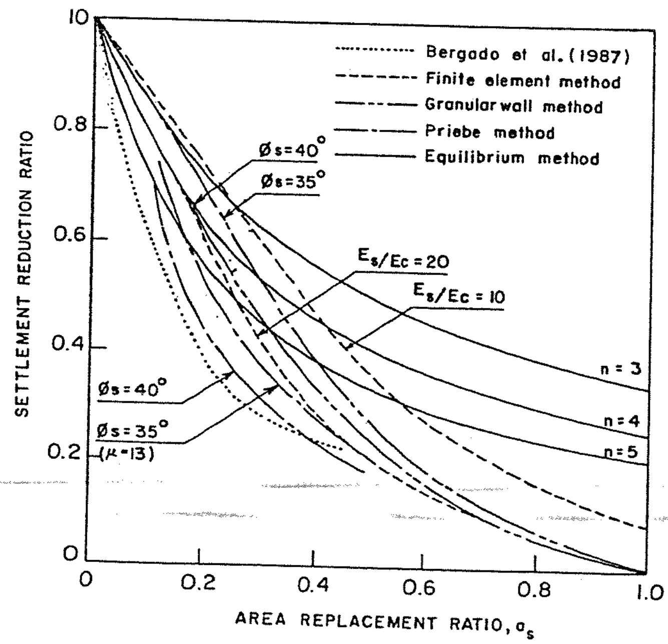

Fig. 5.10 Comparison of Estimating Settlement Reduction of Improved Ground (Aboshi and Suematsu, 1985)

of slices is recommended (Fredlund and Krahn, 1977; Whitman and Bailey, 1967). The method of stability analysis is extended, as an approximation, to evaluate the stability over large areas improved with granular piles and imposed with heavy loads. Stability analysis are usually carried out with the implementation of computer programs. The three general techniques used in stability analysis of the composite ground consist of the profile method, the average shear strength method, and the lumped parameter method, described as follows (see Barksdale and Bachus, 1983):

5.7.1 Profile Method

In the profile method, each row of the granular piles is converted into an equivalent, continuous strip with width, w, as shown in Fig. 5.11. Each strip of granular and cohesive soils is then analyzed using its actual geometry and material properties. For an economical design, the stress concentration developed in the piles must be taken into consideration. The stress concentration in the granular pile results in an increase in resisting shear force. The effect of the stress concentration is being handled by placing thin, fictitious strips of soil above the in-situ soil and granular piles at the embankment interface (see Fig. 5.11). The weight of the fictitious strips of soil placed above the granular piles is relatively large to cause the desired stress concentration when added to the stress caused by the embankment. The weight of the fictitious soil placed above the in-situ soil must be negative to give proper reduction in stress when added to that caused by the embankment. The fictitious strips placed above the in-situ soil and granular piles would have no shear strength, and their weights are respectively expressed as:

$$\gamma_f^c = \frac{(\mu_c - 1)\gamma_1 H'}{\bar{T}} \tag{5.17}$$

$$\gamma_f^s = \frac{(\mu_s - 1)\gamma_1 H'}{\bar{T}} \tag{5.18}$$

where μ_c and μ_s are the stress concentration factors of the in-situ soil and granular piles, respectively, and the other terms are defined as indicated in Fig. 5.11. It must be noted that limits should be imposed on the radius and/or grid size of circle centers so that the critical circle should not be controlled by the weak, fictitious interface layer.

5.7.2 Average Shear Strength Method

The average shear strength method is widely used in stability analysis for sand compaction piles (Aboshi et al. 1979; Barksdale, 1981). The method considers the weighted average material properties of the materials within the unit cell (Fig. 5.12). The soil having the fictitious weighted material properties is then used in stability analysis. Since average properties can be readily calculated, this approach is appealing for both hand and computer calculations.

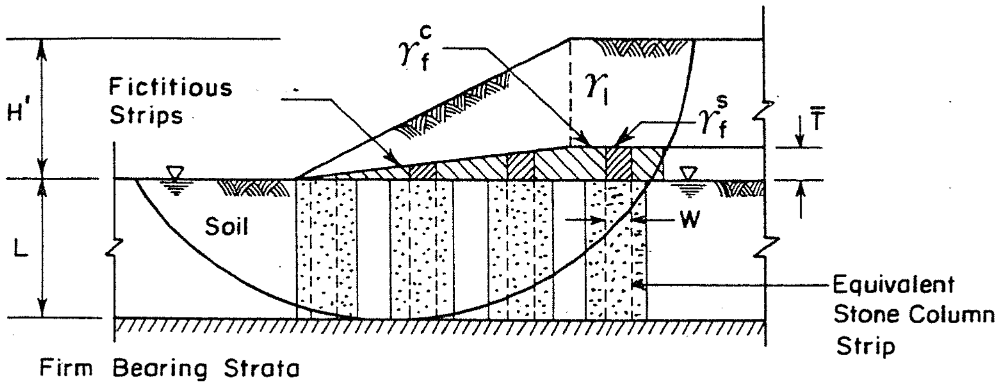

Fig. 5.11 Granular Pile Strip Idealization and Fictitious Soil Layer for Slope Stability Analysis (Barksdale and Bachus, 1983)

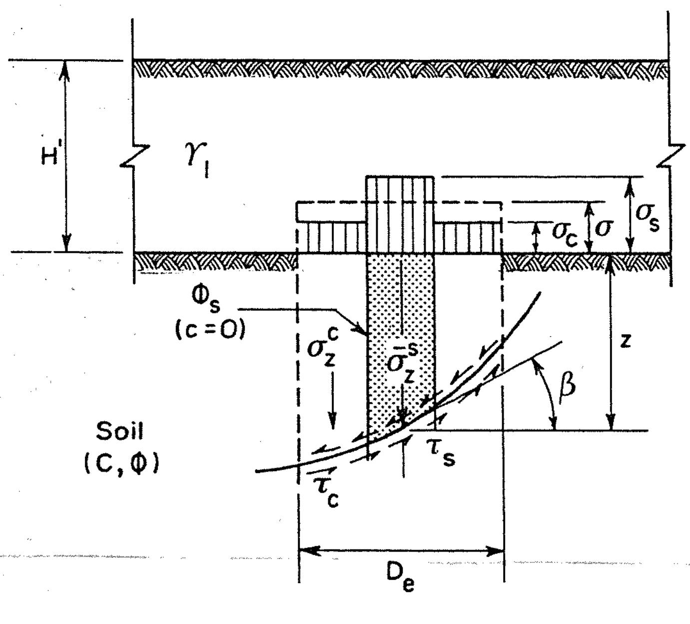

Fig. 5.12 Average Stress Method of Stability Analysis (Barksdale and Bachus, 1983)

However, average properties cannot be generally used in standard computer programs when stress concentration in the granular piles is considered in the analysis.

When stress concentration is considered, hand calculation is preferred. Within the unit cell, the granular pile has only internal friction, ϕ_s, and the surrounding soil is undrained but has cohesion, c, and internal friction, ϕ_c. The state of stresses within the unit cell is also shown in Fig. 5.13. The effective stresses in the granular pile and the total stress in the surrounding soil are respectively expressed as:

$$\sigma_z^s = \gamma_s z + \sigma\mu_s \tag{5.19}$$

$$\sigma_z^c = \gamma_c z + \sigma\mu_c \tag{5.20}$$

where γ_s = bouyant weight of the granular materials; γ_c = saturated unit weight of the surrounding soil; z = depth below the ground surface; σ = stress due to embankment loading, and the other terms are already defined. The shear strength of the granular pile and the surrounding cohesive soil are:

$$\tau_s = \left(\sigma_z^s \cos^2\beta\right)\tan\phi_s \tag{5.21}$$

$$\tau_c = c + \left(\sigma_z^c \cos^2\beta\right)\tan\phi_c \tag{5.22}$$

where ß = inclination of the shear surface with respect to the horizontal. The average weighted shear strength within the area tributary to the granular pile is:

$$\tau = \left(1-a_s\right)\tau_c + a_s\tau_s \tag{5.23}$$

The weighted average unit weight, γ_{avg}, within the composite ground used in calculating the driving moment is:

$$\gamma_{avg} = \gamma_s a_s + \gamma_c a_c \tag{5.24}$$

where γ_s and γ_c are the defined previously. In this approach, the weighted shear strength and unit weight are calculated for each row of granular piles and then used in conventional hand calculation.

When stress concentration is not considered, as in the case of some landslide problems, a standard computer analysis employing average strengths and unit weights, can be performed using a conventional computer program. Neglecting the cohesion in the granular materials and the stress concentration, the shear strength parameters for use in the average shear strength method are:

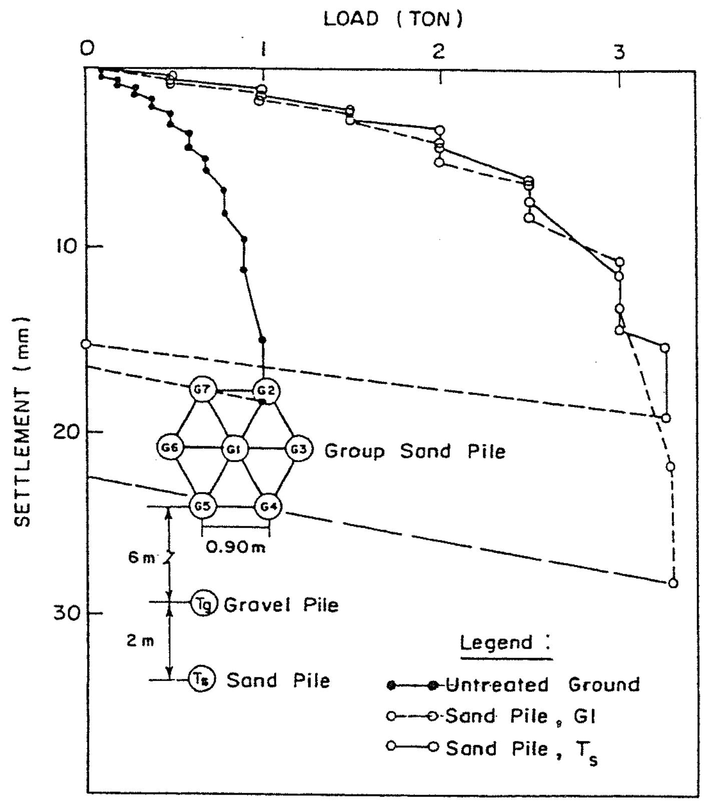

Fig. 5.13 Load-Settlement Relationship of Granular Piles from Full-Scale Load Tests (Bergado et al., 1984)

$$c_{avg} = c(a_s) \tag{5.25}$$

$$(\tan\phi)_{avg} = \frac{\gamma_s a_s \tan\phi_s + \gamma_c a_c \tan\phi_c}{\gamma_{avg}} \tag{5.26}$$

where γ_{avg} is given by Eq. (5.24) using bouyant weight for γ_s and saturated weight for γ_c. DiMaggio (1978) reported that the use of $(\tan\phi)_{avg}$ based just on the area ratio is not correct as can be demonstrated by considering the case when $\phi_c = 0$. If averages based on the area were used, then:

$$(\tan\phi)_{avg} = a_s \tan\phi_s \tag{5.27}$$

The above expression would be appropriate to use if $\gamma_{avg} = \gamma_s$, but incorrect if γ_{avg} is used to calculate the driving moment.

5.7.3 Lumped Parameter Method

The lumped parameter method can be used to determine the safety factor of selected trial circles by either hand calculations or with the aid of a computer. The general approach is described by Chambosse and Dobson (1982). The safety factor of the composite ground is calculated by:

$$SF = \frac{(RM + \Delta RM)}{(DM + \Delta DM)} \tag{5.28}$$

where RM = resisting moment, DM = driving moment, ΔRM = excess resisting moment due to granular piles, ΔDM = excess driving moment due to granular piles. The DM and RM are first calculated for the condition of unimproved ground. Then ΔRM and ΔDM are added to the previously calculated moments, RM and DM, respectively. The approach is generally suited for hand calculations. The use of computer programs is possible only when adding ΔRM and ΔDM which could be calculated by hand.

5.8 RATE OF PRIMARY CONSOLIDATION SETTLEMENT

Previous studies assumed that granular piles could accelerate the consolidation process in the same way as sand drains. In a cohesive soil reinforced with granular piles, pore water moves toward the pile in a curved path having both vertical and radial components of flow. The average degree of primary consolidation could be handled by considering the vertical and radial consolidation effects separately, as expressed by the following equation (Carillo, 1942):

$$U = 1 - (1 - U_z)(1 - U_r) \tag{5.29}$$

where U = average degree of consolidation of the cohesive layer considering both vertical and radial drainage, U_z = degree of consolidation considering only vertical flow, and U_r = degree of consolidation considering radial flow. The degree of consolidation in the vertical direction is calculated by Terzaghi's one-dimensional theory, while that in the radial direction is calculated by Barron's theory. The primary consolidation settlement at any time, t, is expressed as:

$$S_c(t) = U(S_{cf}) \tag{5.30}$$

where $S_c(t)$ = primary consolidation settlement at any time, t; S_{cf} = final primary consolidation settlement.

5.9 STRENGTH INCREASE OF CLAY DUE TO CONSOLIDATION

The rate of construction of embankments on ground improved with granular piles is frequently controlled to allow the shear strength to increase so that the required safety factor against instability is maintained. The undrained shear strength of a normally consolidated clay has been found to increase linearly with effective overburden pressure (Leonards, 1962). Consolidation results in an increase in effective stress due to the dissipation of pore pressure. For a cohesive soil having a linear increase in shear strength with effective stress, the increase in undrained shear strength, $\Delta c(t)$, with time due to consolidation can be expressed as:

$$c(t) = k_1[\sigma \mu_c][U(t)] \tag{5.31}$$

where $k_1 = c/\sigma$, constant of proportionality defining the linear increase in shear strength with effective stress; U(t) = degree of consolidation of the clay at any time, t, and the other terms are defined previously. The ratio of the measured increase in undrained shear strength to the calculated ones is a function of area replacement ratio and stress concentration factor.

5.10 SECONDARY SETTLEMENT

Secondary settlement is important in organic soils and some soft clays, especially with the presence of free draining granular piles wherein primary consolidation is thought to occur in a short period of time. The prediction for secondary settlement is based on the work of Mesri (1973) and can be calculated by:

$$S_s = C_a H \log_{10}\frac{t_2}{t_1} \tag{5.32}$$

where S_s = secondary compression of the layer; C_α = physical constant evaluated by continuing a one-dimensional consolidation test past the end of primary consolidation for a suitable load increment; H = thickness of compressible layer; t_1 = time at the beginning of secondary compression (the time corresponding to 90% of primary consolidation is sometimes used); t_2 = time at which the value of secondary settlement is desired.

5.11 FULL-SCALE LOAD TESTS ON GRANULAR PILES

Initial full-scale load tests on granular piles were performed to study the feasibility of improving the soft Bangkok clay (see Bergado et al. 1984). The results indicated that the granular piles increased the bearing capacity more than 3 to 4 times that of the untreated ground. Further, the adjacent piles acted independently provided that the pile spacing is 3 times the pile diameter or greater, as shown in Fig. 5.13. An investigation of the behavior of granular piles with different densities and different proportions of gravel and sand on soft Bangkok clay was carried out by Bergado and Lam (1987). Table 5.3 shows that for the same granular materials, the ultimate bearing capacity increases with number of blows per layer because of the increase in the densities and friction angle. Using different proportions of gravel and sand shown in Table 5.3, the resulting load-settlement curves are compared in Fig. 5.14 and indicated a higher ultimate pile capacity for pure gravel. The average deformed shape of the granular piles is typically bulging type. It was observed that the maximum bulge occurred near the top of the pile and ranged from 10 cm to 30 cm below the ground surface. With an initial pile diameter of 30 cm, the measurements of bulge are in close agreement with the observations of Hughes et al. (1975), wherein the maximum bulge occurred near the ground surface at a depth approximately equal to one-half to one pile diameter. The results of full-scale load tests on granular piles with different sizes of plates were reported by Bergado et al. (1987). As expected, the settlement decreased as the size of the plate increased for the same amount of total load. It was also indicated that as the ratio D_e/D increased, the settlement of the treated ground approaches the settlement of untreated ground. This implies that beyond D_e/D = 4.0, the settlement of treated ground is almost the same as the settlement of untreated ground. It should be noted that D_e is directly related to the spacing of granular piles depending on the pattern used. The variations of stress concentration in granular piles and clay with D_e/D are shown in Fig. 5.15.

5.12 TEST EMBANKMENT ON GRANULAR PILES

A full-scale test embankment 2.4 m high was constructed by Sim (1986) on a granular pile-improved foundation, and was raised to a height of 4.0 m by Panichayatum (1987) to provide a meaningful basis of comparison with the performance of the nearby 4.0 m high test embankment constructed on Mebra vertical drain-improved foundation by Singh (1986). The performances of these two embankments have been already reported by Bergado et al. (1988,1990a). An evaluation by back-analysis of the geotechnical parameters of the foundation soil improved by granular piles and vertical drains was carried out by Enriquez (1989).

Table 5.3 Properties of Granular Piles (Bergado and Lam, 1987)

Group	1			2			3			4		5	
No. of pile	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12	G13
Proportion of sand in volume	1.0			1.0			1.0			0.3		0.0	
Proportion of gravel in volume	0.0			0.0			0.0			1.0		1.0	
Blows per compacted layer	20			15			10			15		15	
In-situ average density (t/m)	1.73	1.71	1.66	1.64	1.51	1.67	1.47	1.53	1.50	1.91	1.96	1.76	1.79
Average	1.70 t/m^3			1.61 t/m^3			1.50 t/m^3			1.94 t/m^3		1.74 t/m^3	
Friction angle (degree)	39.1	38.4	37.2	37.0	36.0	37.6	35.1	36.2	35.6	37.4	37.9	42.5	44.7
Average	38.2^o			36.9^o			35.6^o			37.7^o		43.3^o	
Ultimate Load (tons)	3.50	3.25	3.25	3.25	3.00	3.00	2.25	2.25	2.00	3.25	3.00	3.50	3.75
Average	3.33 tons			3.08 tons			2.17 tons			3.13 tons		3.63 tons	

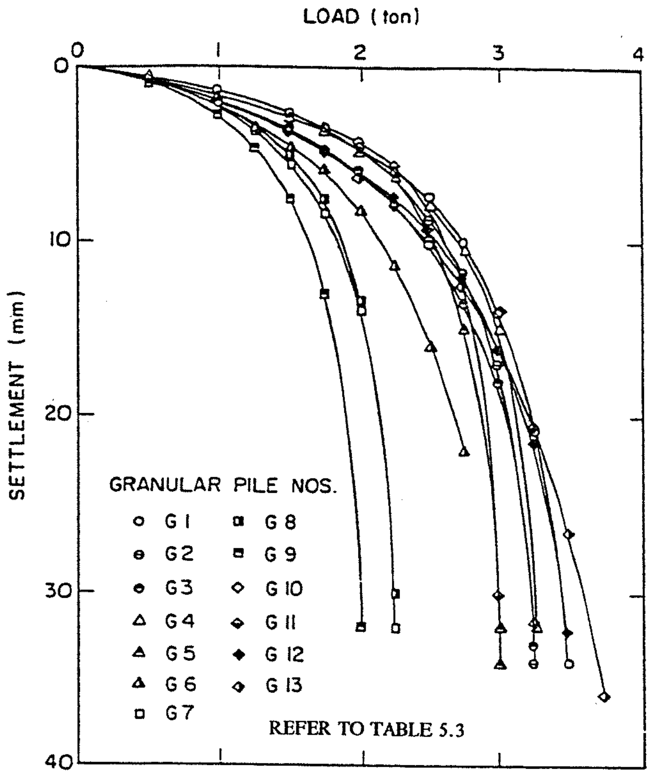

Fig. 5.14 Comparison of Load-Settlement Curves Between Piles with Different Number of Blows per each Compacted Layer During Installation (Bergado and Lam, 1987)

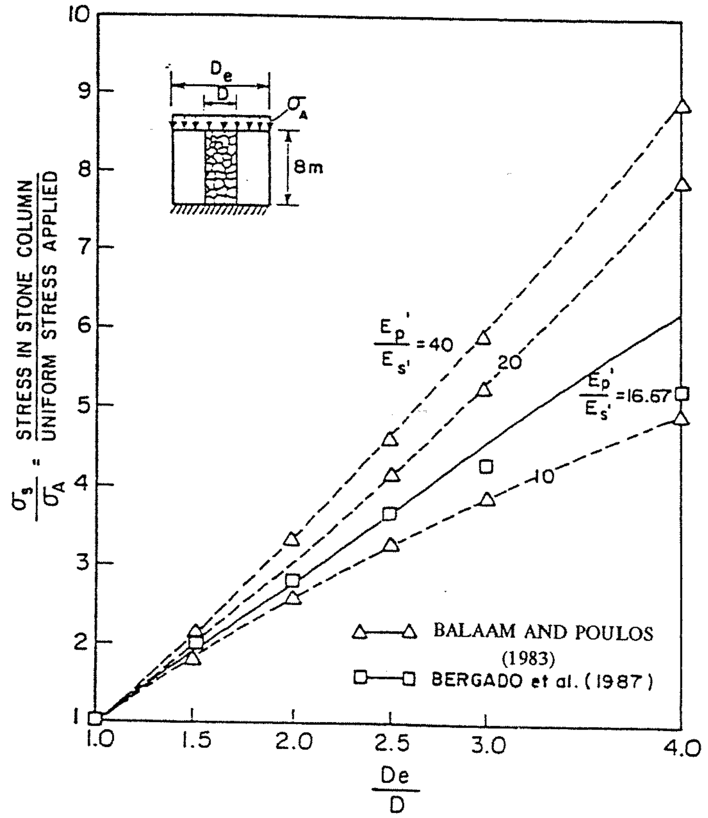

Fig. 5.15 Variation of Vertical Stress in Granular Pile with De/D (Bergado et al. 1987)

The site is located at the Asian Institute of Technology (AIT) campus, about 42 km north of Bangkok at the Central (Chao Phraya) Plain of Thailand. The soil profile at the site, together with the index and strength properties of the soil, is given in Fig. 5.16. The soil profile consists of 2 to 3 m of weathered marine clay underlain by 6 to 8 m thick of soft marine clay and followed by a stratum 5 to 6 m thick of stiff clay layer. The groundwater table fluctuated with the season and varied from 0.5 to 2.0 m below the ground level.

The test embankment had a first stage height of 2.4 m and subsequently was raised to a second stage height of 4.0 m after 345 days. The plan and cross-section of the embankment including the layout of the monitoring instruments are illustrated in Fig. 5.17. The monitoring program included surface and subsurface settlement measurements, pore pressures, vertical earth pressures, and lateral movement. The instrumentations installed consisted of 3 surface settlement plates, 4 subsurface settlement gauges at 1.5 m, 3.0 m, 6.0 m, and 8.0 m depths, 8 closed hydraulic piezometers, 2 earth pressure cells, an SIS Geotechnica C412 type inclinometer casing with readings provided by the SIS Geotechnica sensor, and three lateral movement stakes. The embankment was compacted in layers by a light vibrating plate tamper and was found to have an average density of 18.0 kN/m^3.

The Cased Borehole Method was employed in constructing the granular piles (Bergado et al. 1984; Bergado and Lam, 1987). The compaction was done by means of dropping the 1.6 kN hammer at 0.6 m falling height to the steel disc which was placed on the surface of the granular material. Each layer of the pile was compacted with 15 blows per layer, with a compacted thickness of about 0.6 m. The friction angles of the compacted granular piles obtained from direct shear tests varied from 39 to 45 degrees with compacted densities ranging from 17 to 18.1 kN/m^3. The compacted granular piles were arranged in a triangular pattern with a spacing of 1.5 m. The piles were 30 cm in diameter and have a length of 8.0 m, fully penetrating the soft clay layer. The granular materials consist of whitish-gray, poorly graded crushed limestones with a maximum size of 20 mm. A drainage blanket of 0.25 m thick consisting of clean sand was laid on top of the compacted granular piles.

5.13 PERFORMANCE OF GRANULAR PILES UNDER EMBANKMENT LOADING

Based on full-scale load tests on granular piles, Bergado et al. (1984, 1987) and Bergado and Lam (1987) reported that the bearing capacity of the soft Bangkok clay using granular piles increased by up to 4 times, the total settlements reduced by at least 30%, and the slope stability safety factor increased by at least 25%. The comparative study of the performance of test embankments on granular piles and on vertical drains indicated that the embankment on granular piles settled about 40% less than the embankment on vertical drains, as shown in Fig. 5.18. The study confirms the idea that granular piles function as a reinforcement to the clay rather than drains (Bergado et al. 1988), attributed perhaps to the larger zone of disturbance (smeared zone) surrounding the pile caused by the method of installation.

As mentioned in the previous chapters, using the finite element method (FEM), Long (1992) analyzed the consolidation due to the embankment load by 2-D model and evaluated the

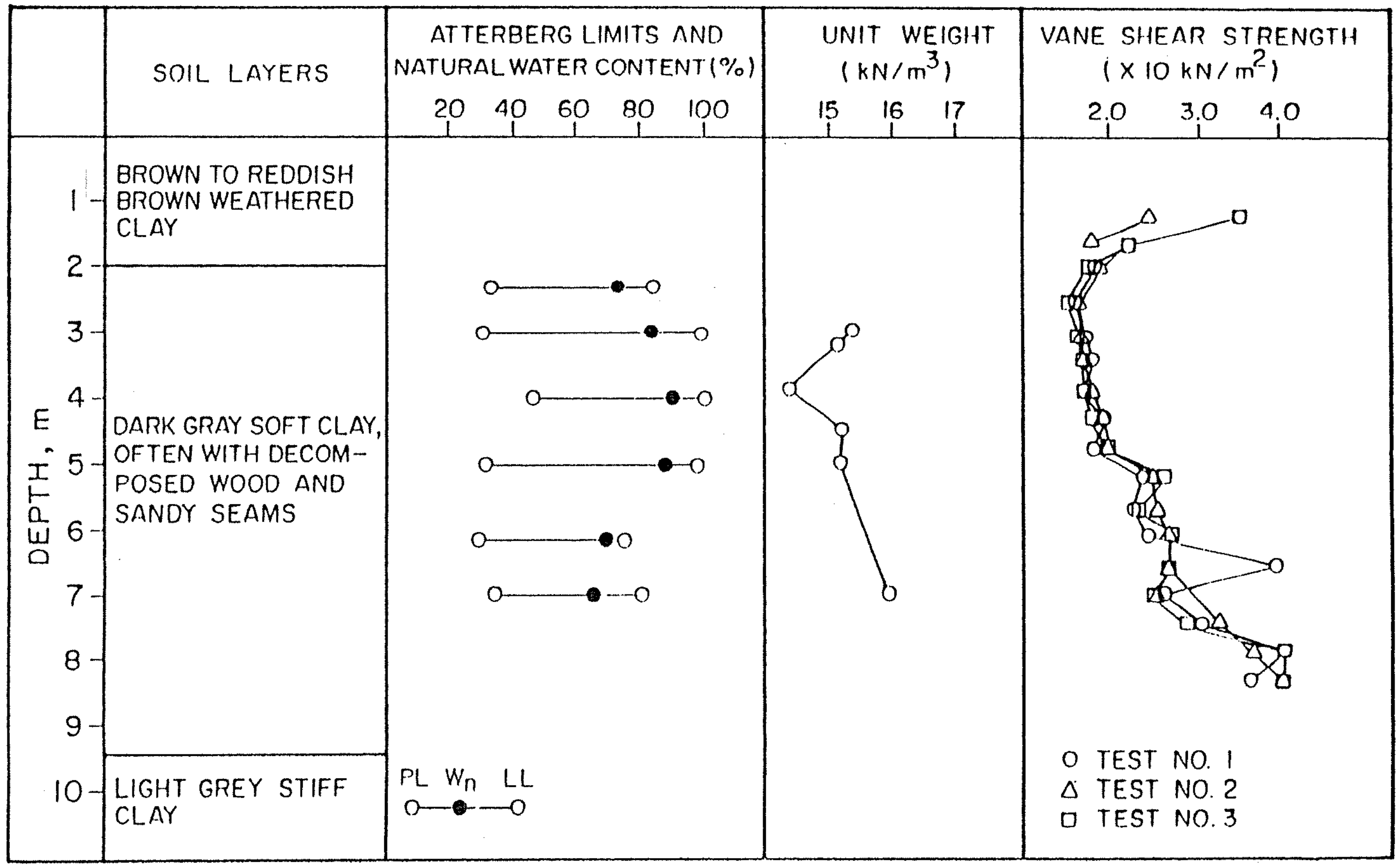

Fig. 5.16 Soil Profile and Properties at AIT Campus